DESIGN OF A SPAN STEEL SPANDREL - BRACED TWO - HINGED ARCH.

BY

C. W. BINDER
O. R. KELLNER
L. A. SIMONS

ARMOUR INSTITUTE OF TECHNOLOGY

1911

DESIGN OF A

210' SPAN STEEL SPANDREL-BRACED

TWO-HINGED ARCH.

A THESIS

Presented by :-

C.W. Binder.

O.P. Kellner

L.A. Simons.

To the

PRESIDENT AND FACULTY

of

ARMOUR INSTITUTE OF TECHNOLOGY

For the Degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

Having Completed the Prescribed Course of Study in

CIVIL ENGINEERING

May 15, 1911.

Alfred E. Phillips
Prof Civil Engineering

H.M. Raymond
Dean of Eng. Studies

L.C. Monin
Dean of Cultural Studies.

The Two-Hinged Spandrel-Braced Steel Arch.

It is customary in the study of arched structures to classify them all under one of three heads, according to the number of hinges they have; therefore, we have the three-hinged arch with two abutment hinges and one at the crown, the two-hinged arch with only the two abutment hinges, the one-hinged arch with a hinge at the crown, and the no-hinged arch having, as the name indicates, no hinges. The last two named types of arches, however, have found but little application in the engineering practice of recent years, and as a consequence have not reached the development attained by the two and three-hinged arches.

The main feature in distinguishing an arched structure from a simple truss or beam is in the matter of reactions. The simple truss under vertical loads has vertical reactions provided one end is so arranged as to permit lateral movement due to deflcetion of truss and to temperature changes; but when the abutments are fixed so as to prevent this lateral movement at the supports, the truss comes under the head of arched structures with reactions which are no longer vertical, being, as they are, in the nature of outward thrusts on the abutments.

In selecting the two-hinged type of arch for study it is necessary to go into a further classification of them, so we divide them into the arch-rib type and the spandrel-braced type. In the former the arch rib alone is subjected to the arch action, the panel loads being applied directly to the rib in such a manner that the part above the rib takes no part in resisting the bending moments and shears.

Another type of the arch rib, also known as a "braced arch", is found in the arch truss consisting of two curved parallel chords connected by diagonal bracing. This style is sometimes confused with the spandrel braced arch of the kind to be described more in detail in the following pages, i. e., the type having a straight horizontal upper chord and an arched lower chord connected by vertical and diagonal bracing. In this arch each and every member of the structure assist in resisting the action of applied loads, at least under most conditions of loading.

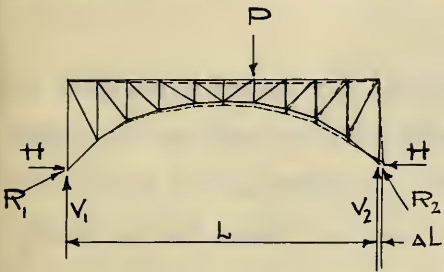
As has been stated the main feature distinguishing the arch from other trussed structures is in the matter of its reactions, which we find may be resolved into vertical and horizontal components - the latter being known as the "horizontal thrust". Thus we find that the arch must be designed to resist stresses due to vertical forces, as in a simple truss, and also to resist stresses due to this horizontal thrust which is caused by deflection of arch and changes in temperature.

In an arch having three hinges this horizontal thrust is easily determined from the simple conditions of static equilibrium. Since the bending moments at the hinges are known to be zero, by taking moments about the center hinge we can write equations in terms of loads and reactions which when equated to zero can be solved for the values of the vertical reactions and horizontal thrust. The two-hinged arch, on the other hand, does not supply a sufficient number of equations of static equilibrium from which to determine these values, so recourse must be had to some other method. This method, as we shall see, is based on either the "elastic theory" or the principle of "least work". The two best known and widely

used application of these two theories are found in methods outlined by Professor Charles E. Greene in his book on "Trusses and Arches, Part III" and in Professors Mansfield Merriman and Henry S. Jacobys' book on "Roofs and Bridges, Part IV". The method as given in Professor Greene's book will be used in the solution of the problem in hand.

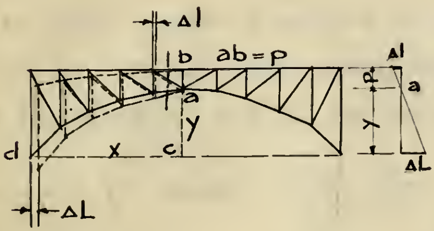
Derivation of Formula for the Horizontal Thrust.

Referring to the sketch given below, first consider the arch fixed at the left abutment but free to move laterally at the right abutment, this condition being indicated by the full lines. Then, under application of load P, changes in lengths of the members of the arch will be produced, thus causing the arch to deflect and the free hinge to be pushed outward as indicated by the dotted lines.



Now if a horizontal force by applied to this free end and of a magnitude sufficient to cause the arch to resume its original position - as shown in full lines - we will have duplicated the stresses in the arch which would be present under application of load P while the hinges are prevented from spreading. In order to obtain the value of this horizontal force necessary to prevent the spreading of the abutment hinges, we first must get the stresses in the members due to a certain loading and then determine the deformations in the members due to this loading. Having these we are enabled, thru an application of the principle of instantaneous centers to find what would be the deformation or movement of the hinges at abutments. From the elastic deformation method, or application of Hooke's Law,

we obtain the expression, $E = \frac{Tl}{A\Delta l}$ (1) where E is the modulus of elasticity, T the total stress in the member, l the length of the member in inches, A the area of the member, and Δl the deformation in the member due to stress T.



In the accompanying diagram let

x = the distance of the center of moments from left abutment as "cd".

Y = the distance of center of moments above springing line of arch, as "ac".

p = lever arm of member, as "ab".

ΔL = horizontal displacement of arch.

Δl = deformation in member.

As previously stated the total horizontal movement of arch at abutments may be considered as made up of the sum of the separate deformations of the members. To consider the effect of change of length in one member to total change of length of arch span, pass a plane cutting three members of arch as shown in above sketch; then draw two of them to an intersection and we get from the principle of instantaneous centers as outlined in any text on Kinematics, the expression $\frac{\Delta L}{\Delta l} = \frac{Y}{p}$ (2) or, to express it in words, the amount of deformation in member is to the total deformation of arch as distance of member from this instant center is from the abutment hinge.

Now let P = vertical force acting upward at abutment

H = horizontal thrust at abutment

t = stress produced in member by H

t' = stress produced in member by P

$T = t + t'$, or total stress in member.

Taking moments about "a", we get,

$$t \times ab = H \times ac \qquad \text{or} \qquad t = \frac{H \times ac}{ab} \qquad (3)$$

$$t' \times ab = P \times cd \qquad \text{or} \qquad t' = \frac{P \times cd}{ab} \qquad (4)$$

Now in order to make equations (3) and (4) more general, substitute for ab, ac, and ad, their equal values p, y, and x, respectively. Then equations (3) and (4) may be written $t = \frac{H.y}{p}$ and $t' = \frac{P.x}{p}$

Also $T = t + t'$ equals $T = \frac{Hy}{p} + \frac{Px}{p}$ or $T = \frac{Hy \quad Px}{p}$ (5)

From (1) $\Delta L = \frac{y}{p} \Delta l$ and from (2) $\Delta l = \frac{TL}{AE}$. $\therefore \Delta L = \frac{y}{p} \cdot \frac{Tl}{AE}$.

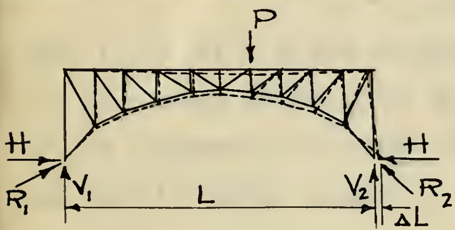
From (5) and (6) $\Delta L = \frac{y}{p} \cdot \frac{l}{AE} \times \frac{Hy + Px}{p} = \frac{1}{AE}\left(\frac{Hy^2}{p^2} + \frac{Pxy}{p^2}\right)$.

Calculating this value of ΔL" for every member of the arch, and adding them together gives for the total horizontal displacement of arch $\Delta L = \frac{1}{AE}\left(\sum \frac{Hy^2}{p^2} + \sum \frac{Pxy}{p^2}\right)$. Since the construction of the arch abutments are such as to prevent this lateral movement or change in length of span, ΔL must be equated to zero. Therefore, in solving the above equation for H, we get $H = \dfrac{\sum P \frac{xyl}{p^2 AE}}{\sum \frac{y^2}{p^2} \cdot \frac{l}{AE}}$ (6)

This then is the general formula for determining the horizontal thrust "H", and is applicable to any spandrel-braced arch. We also note that it contains the unknown value "A", the area of the member, so for purposes of preliminary design this will be considered as unity; and since the term $\frac{l}{AE}$ appears in both numerator and denominator of above expression it may be omitted in the preliminary design. Accordingly, the formula to be used for determining the value of the horizontal thrust due to a load at each successive panel point may

be written
$$H = \frac{\sum \frac{Pxyl}{p^2}}{\sum \frac{y^2 l}{p^2}}.$$

The method of determining the horizontal thrust as outlined in the book by Professors Merriman and Jacoby involves in addition to the elastic theory the principle of least work, or the internal work in the members counteracting the work of external forces.



Considering the arch fixed at the left end but free to move laterally at the right, it may, under no load, be represented as shown in full lines in accompanying sketch. Upon the application of a vertical load P, however, it will assume the position indicated by the dotted lines, the right hinge moving outward a distance ΔL. Now if we apply a horizontal force H of sufficient magnitude to bring the arch back to its original position (shown in full lines) we will have placed the arch in identically the position and under the same conditions existing in a two-hinged spandrel-braced arch under action of vertical load or loads. In order to deduce an expression for the value of this displacement ΔL, were the arch free to move laterally, let

P = vertical load on arch

L = length of member in inches

A = area of cross section of member

S' = stress produced in member by vertical load P

T = stress produced in member by horizontal force of unity applied at the abutment.

e = change in length of any member due to force of unity acting horizontally at the abutment.

Δ = total displacement of arch.

Now from Hooke's Law we know that $e = \frac{TL}{AE}$, and the internal work in the member will be equal to $\frac{1}{2}S'e = \frac{\frac{1}{2}S'TL}{AE}$. Hence, for the entire arch the total internal work is $\sum\frac{\frac{1}{2}S'TL}{AE}$. The external work done by this horizontal force of unity acting through the displacement of the arch is equal to $\frac{1}{2}(\Delta.1)$.

Equating these two values of work, the formula reduces to

$$\Delta = \sum\frac{S'TL}{AE} \qquad (8)$$

Now, if we let U be the stress in any member of the arch due to the horizontal thrust H, we will have that $U = H.t.$ Considering e' the deformation on member under stress U, we find that the internal work in member is $\frac{1}{2}Ue'$. But, $e' = \frac{UL}{AE}$. $\therefore \frac{1}{2}Ue' = \frac{\frac{1}{2}U^2L}{AE}$

Equating this to the external work,

$\frac{1}{2}(H\Delta) = \frac{\frac{1}{2}U^2L}{AE}$ or for complete arch $\Delta = \frac{1}{H}\sum\frac{U^2L}{AE}$

Substituting HT for U, formula for Δ reduces to $H\sum\frac{T^2L}{AE}$. (9)

Equating formulae (8) and (9), we get the expression $H = \frac{\sum \frac{S'TL}{AE}}{\sum\frac{T^2L}{AE}}$ from which may be computed the horizontal thrust for any trussed two-hinged arch due to a load P. It is also to be noticed that this is an expression for getting the value of H under any system of loading, providing S' represents the stresses due to that loading. The stresses S' are always calculated from the vertical reactions V_1 and V_2, the same as if the arch were a simple truss. For purposes of a preliminary design the value of A in the above formula is taken as unity. For the sake of comparison it is interesting to note that the formula by Greene for $H = \frac{\sum \frac{Pxy}{p^2}\frac{l}{AE}}{\sum\frac{y^2}{p^2}\frac{l}{AE}}$ may be changed very easily to the form above given by merely substituting for T its equivalent $\frac{Hy}{p}\frac{Px}{p} = \frac{H\sum\frac{Pxy}{p^2}}{H^2\sum\frac{y^2}{p^2}}$ the H dropping out, since value of T is derived by taking H as unity.

The Design.

Among the number of modern steel structures that span the deep gorges and ravines on the Guatemaula Railroad is a three-hinged spandrel-braced steel arch of about 210' span, crossing what is termed the Rio Fiscal. Since the geological formation at this point was found to be ideal for the construction of a two-hinged arch, this site was selected for the arch to be designed according to methods as already outlined. The present structure at this point is a single-track, 3'-6" gauge, deck arch-bridge having provision made for widening to standard gauge at some future date, and is calculated to withstand the stresses resulting from the passage of two $73\frac{1}{2}$ ton Mogul type engines followed by a uniform load of 5000# per foot of bridge.

This same loading was used in our design of a two-hinged arch for this place, it being found that the locomotive gave an excess panel load of 35,000# followed by a live load per panel of 27,000#. From several existing arches of approximately the same span as the one chosen, the dead load per panel was estimated as 20,000#, and final calculations confirmed us in this estimate, the final average dead panel load being 19,000#. In design of arch members the American Railway & Maintenance of Way Association's specifications for railway bridges were used, and for purpose of comparison checked by Cooper's specifications for railway bridges, the two being found to vary but slightly in final results. On plate #1 are to be found all of the data used in the design of the arch as hereinafter given.

The first step in the determination of the horizontal thrust from the formula $H = \dfrac{\Sigma P\frac{xyl}{AE}}{\Sigma \frac{y^2 l}{AE}}$ was to obtain the values of x/p and y/p,

this being accomplished by means of the diagrams shown on plates #2 and #3 and tabulations on plate #4. By multiplying these values of x/p and y/p with "l" we obtain the values of xyl/p, a summation of which is in the numerator of formula (6), this being shown on plate #5. Similarly a summation of values of y l/p, the denominator in formula (6) is tabulated on plate #6.

The next procedure was to obtain the preliminary value of H for a load P on each panel point: results so obtained are tabulated on plate #7. Knowing the values of the vertical reactions and horizontal thrusts for a load at each panel point, a determination of the stresses in the arch members under panel loads of 1,000# was accomplished by the graphical methods illustrated in the diagrams on plates #16, to #20, inclusive, and results on plate #8. Since the dead, live, and excess panel loads were 20,000#, 27,000#, and 35,000#, respectively, it was merely a matter of multiplying the above mentioned stressed by 20, 27, and 35, to obtain the actual stresses in the members under the preliminary values of V and H. The results of this calculation are given on plates #9, #10, and #13.

Owing to the condition that the arch is anchored at the abutments only, while the greater part which is exposed to the action of lateral and wind forces is at a considerable distance above the anchorage, large overturning moments are given rise to, thus producing vertical forces acting downward on one side of the arch and upward on the other, the transfer of these forces taking place through the sway bracing. We find that the distribution of the wind and lateral forces in a two-hinged arch is not strictly determinate, but after a little consideration of the subject we should expect to find the most rigid members taking these stresses; hence,

we calculated that the upper lateral system, with its heavy floor-beams connections and heavy chord members, would carry all of the wind and lateral forces on the upper chord to the end portal and thence to the abutment, thus leaving the lower lateral system to carry to the abutment only the wind loads on the lower part of the arch. In the design of the sway bracing, however, all of the lateral forces on the upper chord were considered as coming down to the lower chord. All wind and lateral forces were considered to act as live loads.

Since the lower chord panel points are not in the same horizontal plane, we find that a load (horizontal) at each panel point produces an overturning moment about the next lower panel point toward the abutment. These overturning couples, however, may be resolved into vertical loads on the arch, thus causing additional stresses in the arch members. The stresses so obtained are shown in table #11.

The design of the upper and lower lateral systems and of sway and portal bracing were accomplished analytically and stresses found are tabulated on plates #26 and #27. On plate #27 are also given the results obtained in the analytical design of the floorbeams and stringers.

Before a final summation of the various stresses could be made, it was necessary in accordance with the specifications to take into account impact stresses, these to be calculated according to the formula, Impact $= \frac{S \cdot 300}{300 + L}$, where S is the live load stress in the member, and L the loaded length of the arch causing this maximum live load stress. These results are given in the table on plate #13.

The preliminary temperature stresses were calculated according to the method given in Higher Structures, Part IV, a method based again on the elastic theory. By means of the displacement diagram, so called because it gives the relative displacements and final positions of the various panel points due to deformations in the stressed members of the arch when all points - except the middle member - are considered free to move, we found that under a rise or fall of temperature of 50 degrees Fahr. the abutment hinges would be thrust outward a distance of 239", assuming for ease of calculation an area of unity for each of the members and an value of 10,000 pounds for the coefficient of elasticity E, and stresses in members those due to a horizontal thrust of 100#. This horizontal thrust can be used in the calculation of the temperature stresses, because it is well known that the effect of changes in temperature on a two-hinged arch is to produce stresses in arch members the same as those caused by a horizontal force applied at abutment hinges. A reduced figure of this displacement diagram is shown on plate #12, the short heavy lines denoting th deformations in members and the light lines the direction of movement and final location of the various panel points with regard to the fixed member LL'. The Actual movement of the hinges, were they free to move laterally, under a rise or fall of temperature of 50 degrees, we found to be 210' x 12" x 50 x .0000065 = .819". Now taking the deformation of 239" obtained under the assumption that E is 10,000 and A unity, we divide it by 3,000 x 26.5, the last figure being the assumed average areas of the members. This gives .0015" total movement of abutment hinges under a horizontal load of 100#. Dividing the amount of movement of hinges occasioned by temperature changes by this value

gives 27,300# as the amount of horizontal thrust required to counteract the effect of temperature changes. Multiplying the constants given in table #14 by this value of H gives the stresses in the arch members due to this cause.

In determining the final temperature stresses a final displacement diagram as shown on plate #12 was constructed. The deformations used in the construction of this diagram were calculated from the usual formula, but using actual areas in place of assumed areas. The final value of horizontal thrust due to temperature was found to be 17,850#, and the final stresses as tabulated on plate #24 and #25.

With all of the preliminary stresses determined, as shown on plate #13, we proceeded with the design of the arch members. In determining these preliminary areas, as well as the final areas, the wind stresses were not taken into consideration unless they amounted to 30% of the sum of the stresses from all other sources. Where they did amount to 30% of the sum of the other stresses, the designing stress was increased 25% over what ordinarily would be allowed, this being as per specifications. The web tension members were designed under an allowable stress of 16,000# per square inch, and members in compression according to the straight line formula $S\ 16{,}000\# - 70\frac{l}{R}$. Where members were found to undergo a reversal of stress during the passage of a train over the structure, 50% of the smaller stress was added to the larger and member designed in keeping with this result.

After having determined our preliminary values of the areas of the members, we proceeded to determine a more accurate value of the horizontal thrusts from loads on the different panel points. It

will be remembered that in obtaining the preliminary values of H, we treated the value of A in the equation $H=\frac{\sum \frac{P x y l}{p^2 A E}}{\sum \frac{y^2 l}{p^2 A E}}$ as unity, Now going back and placing these preliminary values of A in these equations, new values of the horizontal thrusts were obtained as shown on plates #14 and #15. An inspection of the preliminary and last named values of H obtained show that there is but little difference, so little difference in fact that it was not considered necessary to make a third calculation for it.

With these new values of H given on plate #15 the same procedure as has just been outlined was followed, preliminary diagrams corrected in their values of H and new diagrams drawn, from which were scaled the true stresses in the members. The diagrams on plates #16, #17, #18, #19, and #20 show the stresses in the arch members due to loads of 1,000# on panel points 1, 2, 3, 4, and 5, respectively. These values are tabulated on plate #21, and the summation column gives the stresses in members due to a dead panel load of 1,000# on each panel point. The actual dead load stresses, obtained by multiplying the values in the summation column of plate #21 by twenty, are tabulated on plate #24.

As in the preliminary, the values of the live load stresses due to a panel load of 27,000# are obtained by multiplying the constants in table #21 by twenty-seven. These results are tabulated on plate #22, and stresses due to excess panel load of 35,000# are given on plate #23. In determining the maximum stress in a member we placed the excess panel load at the panel point giving the greatest stress in the member, and considered the remaining panel points, causing the same kind of stress, covered with a live load of 27,000#.

The final stresses caused by combination of live, dead, wind, temperature, and impact loads are tabulated on plate #24, and a summary of all stresses together with final design, size, and weights of members are given on plate #25.

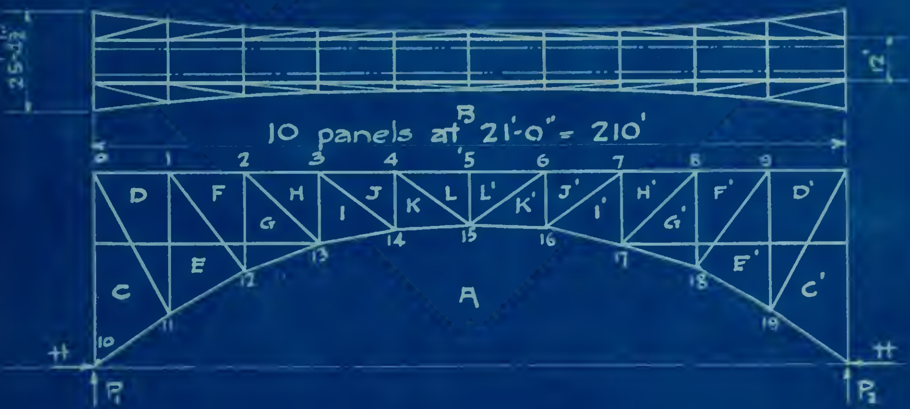
****

****

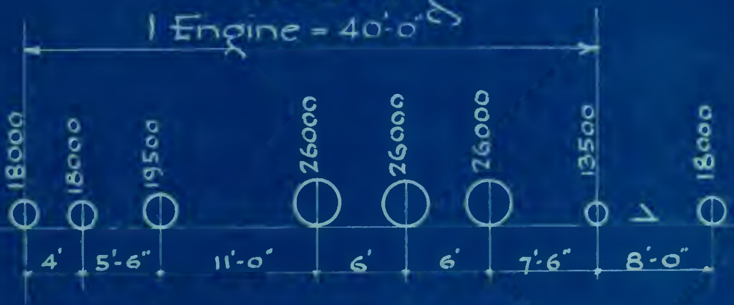
****

Two-Hinged Spandrel Braced Arch

Span = 210'. Depth at Abutment = 53'-6". At center = 15'-0".
Side batter = 1 in 8.



Loading.



Two 73½ Ton Narrow Gauge Mogul Engines
followed by a uniform load of 2500* per ft. of track.
Excess Panel Load = 35000*.
Live " " = 27,000*.
Dead " " = 20,000*. (Assumed)

Diagram giving Stresses in Arch due to a Load of 1000# (applied vertically) at Abutment. These stresses divided by "P" give values of $\frac{x}{p}$.

Scale: 1" = 1000#.

Diagram giving stresses in Arch due to a force of 1000# (applied horizontally) at abutment. These stresses divided by H give values of $\frac{y}{p}$.
Scale: 1" = 400#

Values of $\frac{x}{p}$ for P=1000#
Values of $\frac{y}{p}$ for H=1000#

Upper Chord			Verticals		
Member	$\frac{x}{p}$	$\frac{y}{p}$	Member	$\frac{x}{p}$	$\frac{y}{p}$
BD	- .535	+ .358	BC	- 1.000	+ .665
BF	- 1.452	+ .850	DE	- 1.265	+ .680
BH	- 2.970	+ 1.520	FG	- 1.530	+ .674
BJ	- 5.070	+ 2.215	HI	- 1.660	+ .550
BL	- 7.020	+ 2.549	JK	- 1.375	+ .235
BL'	- 7.020	+ 2.549	J'K'	+ .427	+ .235
BJ'	- 7.622	+ 2.215	H'I'	- .500	+ .550
BH'	- 6.985	+ 1.520	F'G'	- 1.140	+ .674
BF'	- 5.840	+ .850	D'E'	- 1.440	+ .680
BD'	- 4.792	+ .358	B'C'	0	.665

Lower Chord			Diagonals		
Member	$\frac{x}{p}$	$\frac{y}{p}$	Member	$\frac{x}{p}$	$\frac{y}{p}$
AC	0	- 1.200	CD	+ 1.13	- .755
AE	+ .6	- 1.520	EF	+ 1.565	- .840
AG	+ 1.55	- 1.969	GH	+ 2.150	- .950
AI	+ 3.045	- 2.581	IJ	+ 2.678	- .888
AK	+ 5.09	- 3.223	KL	+ 2.380	- .409
AK'	+ 7.645	- 3.223	K'L'	- .74	- .409
AI'	+ 7.145	- 2.581	I'J'	+ .81	- .880
AG'	+ 6.216	- 1.969	G'H'	+ 1.615	- .950
AE'	+ 5.380	- 1.520	E'F'	+ 1.78	- .840
AC'	+ 4.717	- 1.200	C'D'	+ 1.80	- .755

Values of $\frac{xyl}{p^2}$.

Upper Chord			Verticals		
Member	$\frac{xyl}{p^2}$	$\Sigma \frac{xyl}{p^2}$	Member	$\frac{xyl}{p^2}$	$\Sigma \frac{xyl}{p^2}$
BD	- 4.022	- 4.02	BC	-35.578	- 35.578
BF	-25.918	- 29.94	DE	-34.098	- 69.676
BH	- 94.820	- 124.76	FG	-29.755	- 99.431
BJ	-235.830	- 360.59	HI	-19.319	-118.750
BL	-357.770	- 718.36	JK	- 5.363	-124.113
BL'	-357.770	-1094.13	J'K'	+ 1.660	-122.453
BJ'	-354.480	-1448.61	H'I'	- 5.819	-128.272
BH'	-222.930	-1671.54	F'G'	-22.175	-150.447
BF'	- 110.500	-1782.04	D'E'	-38.815	-189.262
BD'	- 36.030	-1818.07	B'C'	Fixed	

Lower Chord			Diagonals		
Member	$\frac{xyl}{p^2}$	$\Sigma \frac{xyl}{p^2}$	Member	$\frac{xyl}{p^2}$	$\Sigma \frac{xyl}{p^2}$
AC	0	0	CD	- 38.271	- 38.271
AE	- 21.532	- 21.532	EF	- 46.931	- 85.202
AG	- 68.273	- 89.805	GH	- 60.882	-146.084
AI	- 169.190	- 258.995	IJ	- 63.564	-209.648
AK	- 345.210	- 604.205	KL	- 25.132	-234.78
AK'	-518.670	-1122.875	K'L'	+ 7.812	-226.968
AI'	-396.460	-1519.335	I'J'	- 19.226	-246.194
AG'	-273.800	-1793.135	G'H'	-45.728	-291.922
AE'	-193.070	-1986.205	E'F'	- 53.344	-345.266
AC'	-142.420	-2128.625	C'D'	- 60.964	-406.230

Values of $\frac{y^2 l}{p^2}$ & $\frac{y^2 l}{p^2 A}$

Upper Chord			Verticals		
Member	$\frac{y^2 l}{p^2}$	$\frac{y^2 l}{p^2 A}$	Member	$\frac{y^2 l}{p^2}$	$\frac{y^2 l}{p^2 A}$
BD	2.692	.22	BC	23.660	1.135
BF	15.170	1.26	DE	18.301	1.519
BH	48.523	3.30	FG	13.112	1.089
BJ	103.050	5.02	HI	6.401	.532
BL	136.450	5.81	JK	.914	.076
Sum =	305.885	15.61	Sum =	62.388	4.351
	2	2		2	2
Total Σ =	611.77	31.22	Total Σ =	124.776	8.702

Lower Chord			Diagonals		
Member	$\frac{y^2 l}{p^2}$	$\frac{y^2 l}{p^2 A}$	Member	$\frac{y^2 l}{p^2}$	$\frac{y^2 l}{p^2 A}$
AC	36.170	1.229	CD	25.568	2.865
AE	54.532	2.058	EF	25.185	2.820
AG	86.720	3.690	GH	26.910	3.015
AI	143.230	6.970	IJ	21.079	2.365
AK	218.720	11.070	KL	4.318	.484
Sum =	539.372	25.017	Sum =	103.060	11.549
	2	2		2	2
Total Σ =	1078.744	50.034	Total Σ =	206.120	23.098

For entire arch:

$$\Sigma \frac{y^2 l}{p^2} = 611.77 + 1078.744 + 124.776 + 206.12 = 2021.41$$

$$\Sigma \frac{y^2 l}{p^2 A} = 31.22 + 50.034 + 8.702 + 23.098 = 113.054$$

Preliminary Values of H.

"W" on joint #1.

4.022	1782.04
0.0	1986.205
35.578	189.262
38.271	345.266
77.871	4302.773
.9	.1
70.0839	430.2773
	70.0839
	500.3612

$$H = \frac{500.3612}{2021.41} = .24914\,W$$

"W" on joint #2.

29.94	1671.54
21.532	1793.135
69.676	150.447
85.207	291.922
206.350	3907.044
.8	.2
165.080	781.4088
	165.080
	946.4888

$$H = \frac{946.4888}{2021.41} = .46823\,W$$

"W" on joint #3.

124.76	1448.61
89.805	1519.335
99.431	128.272
146.084	246.194
460.080	3342.411
.7	.3
322.056	1002.7233
	322.056
	1324.7793

$$H = \frac{1324.7793}{2021.41} = .6552\,W$$

"W" on joint #4.

360.590	1094.130
258.995	1122.875
118.750	122.453
209.648	226.968
947.983	2566.426
.6	.4
568.7898	1026.5704
	568.7898
	1595.3602

$$H = \frac{1595.3602}{2021.41} = .7892\,W$$

"W" on joint #5.

718.360
604.205
124.113
234.780
1681.458
.5
840.729 × 2 = 1681.458

$$H = \frac{1681.458}{2021.41} = .8318\,W$$

Joint	#1	#2	#3	#4	#5
P_1	.9 W	.8 W	.7 W	.6 W	.5 W
P_2	.1 W	.2 W	.3 W	.4 W	.5 W

Stresses due to a Single Load of 1000# placed at panel points as indicated below. All areas considered equal.

Mem.	Joint #1	#2	#3	#4	#5	#6	#7	#8	#9	Σ
BD	-393	-249	-136	-35	+38	+69	+74	+72	+37	-523
BF	-369	-746	-454	-195	0	+97	+126	+127	+71	-1343
BH	-315	-646	-1072	-572	-186	+30	+118	+147	+89	-2407
BJ	-194	-442	-808	-1265	-638	-248	-45	+64	+57	-3519
BL	-48	-160	-404	-753	-1315	-753	-404	-160	-48	-4045
AC	-300	-577	-795	-958	-1015	-952	-795	-577	-302	-6271
AE	+157	-249	-586	-850	-985	-970	-825	-610	-324	-5242
AG	+127	+291	-222	-642	-898	-954	-841	-639	-343	-4121
AI	+66	+178	+419	-230	-675	-846	-799	-633	-348	-2868
AK	-57	-29	+145	+468	-210	-552	-618	-538	-309	-1700
BC	-734	-469	-258	-67	+70	+125	+138	+119	+67	-1009
DE	-971	-689	-440	-221	-50	+38	+72	+74	+47	-2140
FG	+56	-900	-625	-380	-185	-68	-8	+20	+18	-2072
HI	+86	+163	-789	-551	-350	-220	-130	-67	-25	-1883
JK	+105	+202	+268	-635	-482	-357	-253	-162	-74	-1388
CD	+830	+531	+291	+75	-80	-143	-157	-134	-77	+1136
EF	-36	+850	+543	+273	+65	-50	-90	-91	-58	+1406
GH	-78	-140	+880	+535	+265	+95	+11	-30	-25	+1513
IJ	-149	-261	-340	+886	+575	+353	+208	+107	+40	+1419
KL	-179	-346	-496	-630	+835	+620	+439	+277	+129	-649

Preliminary Live Load Stresses due to Excess Panel Load of 35000#.

Member	Joint #1	#2	#3	#4	#5	#6	#7	#8	#9
BD	-13755	- 8715	- 4760	- 1225	+ 1330	+ 2415	+ 2590	+2520	+ 1295
BF	-12900	-26110	-15900	- 6830	0	+ 3400	+ 4420	+4445	+ 2485
BH	-11000	-22610	-37500	-20000	- 6510	+ 1050	+ 4130	+5145	+ 3110
BJ	- 6800	-15470	-33300	-44300	-22300	-.8700	- 1575	+2240	+ 1995
BL	- 1680	- 5620	-14150	-26400	-46000	-26400	-14150	-5620	- 1680
AC	-16500	-20195	-27800	-33550	-35500	-33000	-27800	-20195	-10590
AE	+ 5500	- 8715	-20500	-29800	-34500	-34000	-28900	-21350	-11340
AG	+ 4450	+10185	- 7770	-22500	-31450	-33400	-29400	-22365	-12000
AI	+ 2310	+ 6230	+14700	- 8050	-23600	-29600	-28000	-22155	-12180
AK	- 1995	- 1015	+ 5080	+16400	- 7350	-19320	-21600	-18830	-10820
BC	-25700	-16315	- 9030	- 2340	+ 2450	+ 4375	+ 4830	+ 4160	+ 2350
DE	-34000	-24115	-15600	- 7730	- 1750	+ 1330	+ 2520	+ 2590	+ 1680
FG	+ 1960	-31500	-21900	-13300	- 6500	- 2380	- 280	+ 700	+ 630
HI	+ 3010	+ 5705	-27600	-19300	-12250	- 7700	- 4540	- 2345	- 876
JK	+ 3780	+ 7070	+ 9380	-22200	-16900	-12500	- 8870	- 5670	- 2590
CD	+29000	+18600	+10200	+ 2630	- 2800	- 5000	- 5500	- 4700	- 3700
EF	- 1220	+29800	+19600	+ 9560	+ 2270	- 1750	- 3150	- 3180	- 2030
GH	- 2730	- 4900	+30800	+18700	+ 9300	+ 3330	+ 385	- 1050	- 875
IJ	- 5220	- 9130	-11900	+31000	+20200	+12350	+ 7280	+ 3750	+ 1400
KL	- 6270	-12100	-17350	-22100	+29300	+21700	+15400	+ 9700	+ 4520

Preliminary Live Load Stresses due to a Panel Load of 27000#

Member	#1	#2	#3	#4	#5	#6	#7	#8	#9
BD	-10610	-6725	-3670	-945	+1025	+1865	+1998	+1945	+999
BF	-9970	-28180	-12250	-5260	0	+2620	+3400	+3430	+1918
BH	-8500	-17450	-29000	-15450	-1525	+810	+3185	+3970	+2405
BJ	-5240	-19920	-21800	-34100	-17250	-6700	-1215	+1730	+1540
BL	-1296	-4320	-10900	-20350	-35500	-20350	-10900	-4320	-1296
AC	-8100	-15600	-21450	-25900	-27500	-25700	-21450	-15600	-8150
AE	+4240	-6725	-15850	-22950	-26600	-26200	-22280	-16480	-8750
AG	+3430	+7850	-6000	-17320	-24250	-25750	-22700	-17250	-9260
AI	+1780	+4800	+11300	-6200	-18250	-22850	-21570	-17100	-9400
AK	-1540	-785	+3910	+12650	-5680	-14900	-16700	-14500	-8350
BC	-19800	-12650	-6970	-1810	+1890	+3380	+3726	+3213	+1809
DE	-26200	-18600	-11900	-5920	-1350	+1025	+1945	+2000	+1270
FG	+1515	-24300	-16900	-10250	-5000	-1835	-216	+540	+486
HI	+2320	+4400	-21300	-14900	-9450	-6940	-3510	-1810	-675
JK	+2835	+5460	+7240	-17150	-1300	-9650	-6835	-4355	-2000
CD	+22410	+14337	+7850	+2025	-2160	-3860	-4240	-3618	-2079
EF	-972	+22950	+14650	+7380	+1755	-1350	-2430	-2457	-1566
GH	-2100	-3780	+23750	+14450	+7150	+2565	+297	-810	-675
IJ	-4023	-7047	-9180	+23950	+15500	+9545	+5620	+2889	+1080
KL	-4833	-9342	-13400	-17000	+22550	+16750	+11840	+7479	+3483

Wind Stresses in Arch Members due to Vertical Loads resulting from Overturning Moments about Lower panel points.

Member	Joint #1	#2	#3	#4	#5	#6	#7	#8	#9
Load	26100#	17300#	10200#	3900#	900#	3900#	10200#	17300#	26100#
BD	-10250	-4310	-1388	-137	+34	+270	+755	+1246	+967
BF	-9625	-12900	-4630	-758	0	+378	+1280	+2200	+1850
BH	-8200	-11170	-10930	-2230	-167	+117	+1222	+2280	+2320
BJ	-5070	-7630	-8230	-4930	-575	-970	-458	+1108	+1490
BL	-1252	-2770	-4120	-2940	-1087	-2940	-4120	+2770	-1250
AC	-7830	-10000	-8100	-3740	-321	-3720	-8100	-10000	-7900
AE	+4100	-4310	-5978	-3320	-885	-3780	-8420	-10570	-7750
AG	+3320	+5030	-2265	-2500	-808	-3720	-8600	-11070	-8950
AI	+1720	+3080	+4276	-897	-608	-3300	-8150	-11000	-9080
AK	-1485	-507	+1480	+1835	-189	-2150	-6320	-9300	-8060
BC	-19180	-8100	-2630	-262	+63	+49	+7100	+2060	+1750
DE	-25300	-11930	-4480	-863	-45	+148	+735	+1280	+1225
FG	+1460	-15550	-6375	-1482	-166	-265	-82	+346	+470
HI	+2180	+2820	-8040	-2150	-314	-858	-1325	-1160	-6525
JK	+2740	+3490	+2730	-2475	-433	-1405	-2580	-2810	-1930
CD	+21700	+9175	+2970	+293	-72	-558	-1600	-2320	-2010
EF	-938	+14700	+5540	+1065	+58	-195	-918	-1580	-1520
GH	-2035	-2422	+8925	+2090	+239	+370	+112	-520	-650
IJ	-3890	-4510	-3470	+3460	+518	+1375	+2120	+1850	+1045
KL	-4675	-6000	-5070	-2460	+752	+2420	+4480	+4800	+3370

Final Diagram.
Values of H, P, L, & E
same as given below. A=
actual area of member.

Preliminary Diagram giving Horizontal Displacement of Arch under a Temperature Load of H=100#. Deformations calculated from formula $\lambda = \frac{PL}{AE}$, where P= stress in member due to H=100#, L= length of member in inches, A= area of unity, E= 10,000.

Preliminary Stresses.

Member	Dead	+ Live	- Live	Impact	Temp.	Wind	Totals With wind.	Totals Without wind
BD	-10460	7828	25115	-19100	± 9780	-16085	-155040	-64455
BF	-26860	11368	53590	-40750	24600	-27913	-300713	-140800
BH	-48140	10370	83925	-60300	41500	-32814	-440679	-233865
BJ	-70380	3270	108425	-70700	60500	-33059	-441064	-310005
BL	-80900	0	119732	-70500	69700	-20480	-568312	-340832
AC	-125420	0	177450	-69200	33800	-59711	-489381	-405870
AE	-104840	4740	153735	-91300	41500	-45013	-482378	-391365
AG	-82420	11280	130180	-85000	53800	-37900	-454600	-351400
AI	-57360	17880	107120	-70300	70500	-33035	-410815	-300780
AK	-34000	16560	67355	-43800	88000	-28000	-343160	-233155
BC	-20180	14018	47130	-36800	18150	-21022	-133282	-112260
DE	-42800	6240	70470	-50600	18550	-42625	-225025	-182420
FG	-41740	2541	65701	-47300	18400	-22920	-196061	-173141
HI	-37660	6720	64875	-42200	15000	-20375	-185210	-164835
JK	-27760	15535	58060	-39800	64200	-11900	-143940	-132040
CD	+22720	53212	21700	+40400	20600	+34150	+171080	+136930
EF	+28120	53585	9490	+40700	22940	+21863	+167168	+145305
GH	+30260	55262	8491	+39800	25900	+11650	+162872	+151222
IJ	+28380	65634	22970	+35000	24250	+10368	+163632	+153264
KL	+12980	68852	49675	+52300	11150	+15822	+144602	+128780

Values of $\frac{xyl}{p^2A}$.

Upper Chord			Verticals		
Member	$\frac{xyl}{p^2A}$	$\Sigma \frac{xyl}{p^2A}$	Member	$\frac{xyl}{p^2A}$	$\Sigma \frac{xyl}{p^2A}$
BD	- .323	- .323	BC	- 1.732	- 1.732
BF	- 2.15	- 2.473	DE	- 2.825	- 4.557
BH	- 6.45	- 8.923	FG	- 2.465	- 7.022
BJ	- 11.47	- 20.393	HI	- 1.603	- 8.625
BL	- 15.21	- 35.603	JK	- .445	- 9.070
BL'	- 15.21	- 50.813	J'K'	+ .138	- 8.932
BJ'	- 17.25	- 68.063	H'I'	- .483	- 9.415
BH'	- 15.18	- 83.243	F'G'	- 1.835	- 11.250
BF'	- 9.15	- 92.393	D'E'	- 3.215	- 14.465
BD'	- 2.99	- 95.383	B'C'	-	Fixed

Lower Chord			Diagonals		
Member	$\frac{xyl}{p^2A}$	$\Sigma \frac{xyl}{p^2A}$	Member	$\frac{xyl}{p^2A}$	$\Sigma \frac{xyl}{p^2A}$
AC	0	0	CD	- 4.29	- 4.29
AE	- .813	- .813	EF	- 5.25	- 9.54
AG	- 2.90	- 3.703	GH	- 6.82	- 16.36
AI	- 8.23	- 11.933	IJ	- 7.12	- 23.48
AK	- 17.41	- 29.353	KL	- 2.82	- 26.30
AK'	- 26.15	- 55.503	K'L'	+ .875	- 25.425
AI'	- 19.29	- 74.793	I'J'	- 2.155	- 27.58
AG'	- 11.64	- 86.433	G'H'	- 5.12	- 32.70
AE'	- 7.30	- 93.733	E'F'	- 5.98	- 38.68
AC'	- 4.84	- 98.573	C'D'	- 6.83	- 45.51

Final Values of "H."

"W" on joint #1.

.323	92.39
.0	93.73
1.732	14.465
4.29	38.68
6.345	239.265
.9	.1
5.7105	23.9265
	5.7105
	29.637

$$H = \frac{29.637}{113.054} = \underline{.26215\ W}$$

"W" on joint #2

2.473	83.24
.813	86.43
4.557	11.25
9.54	32.70
17.383	213.62
.8	.2
13.9064	42.724
	13.9064
	56.6304

$$H = \frac{56.63}{113.054} = \underline{.50091\ W}$$

W on joint #3.

8.923	68.06
3.70	74.79
7.022	9.415
16.36	27.58
36.005	179.845
.7	.3
25.2035	53.9535
	25.2035
	79.1570

$$H = \frac{79.157}{113.054} = \underline{.70017\ W}$$

W on joint #4.

20.39	50.81
11.93	55.50
8.65	8.932
23.48	25.425
64.45	140.667
.6	.4
38.67	56.2668
	38.67
	94.9368

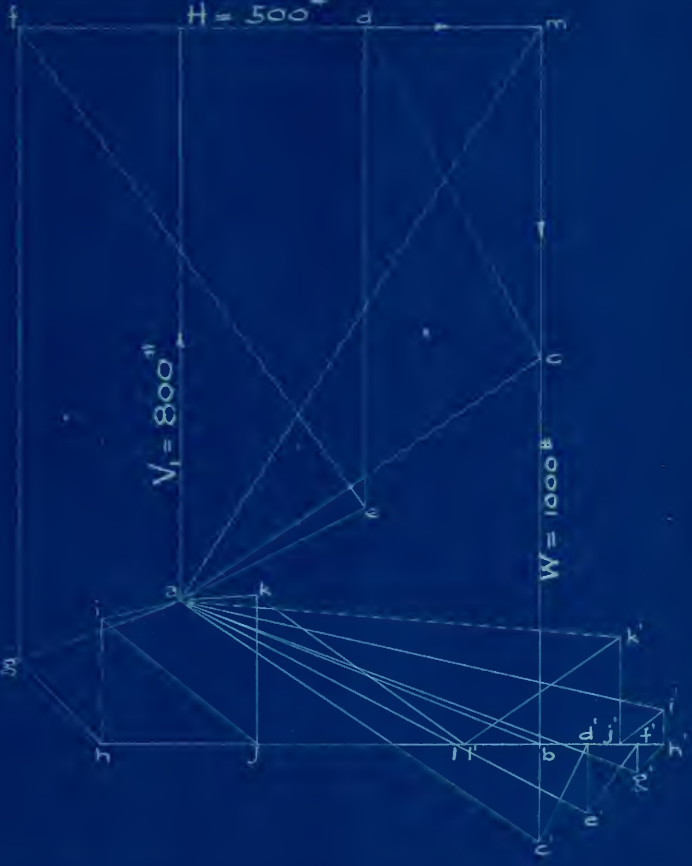
$$H = \frac{94.9368}{113.054} = \underline{.83974\ W}$$

W on joint #5.

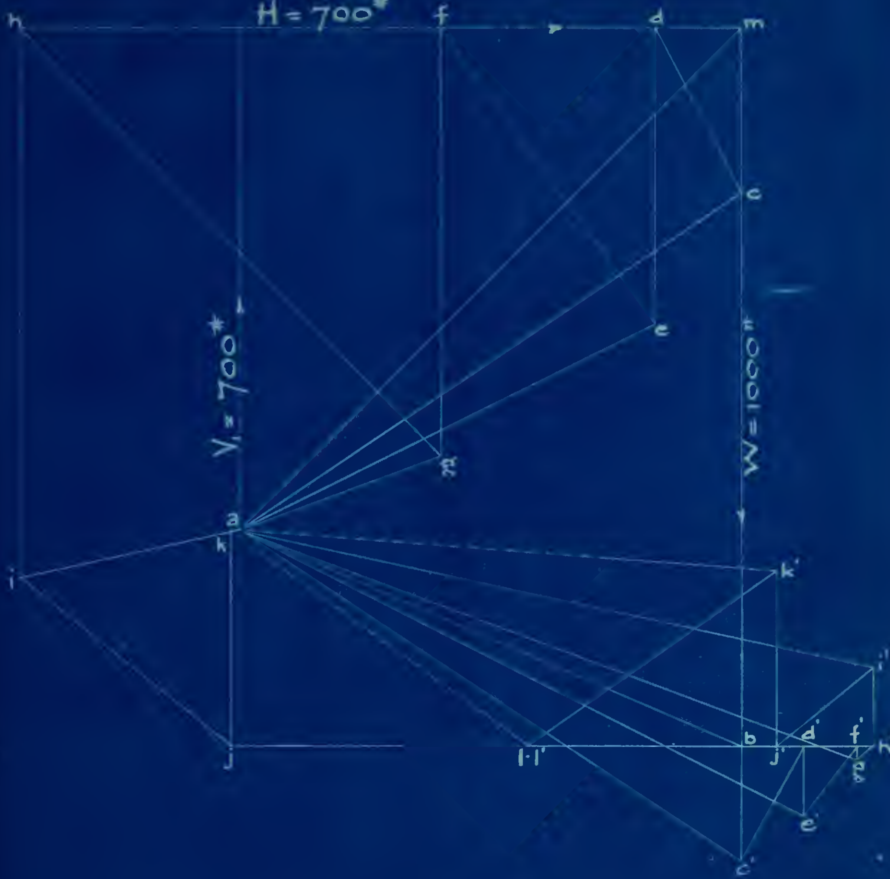
35.60
29.35
9.07
26.30
100.32
.5
50.16 × 2 = 100.32.

$$H = \frac{100.32}{113.054} = \underline{.88126\ W}$$

Stress Diagram. Load of 1000# on joint #1.
Scale: 1" = 150#

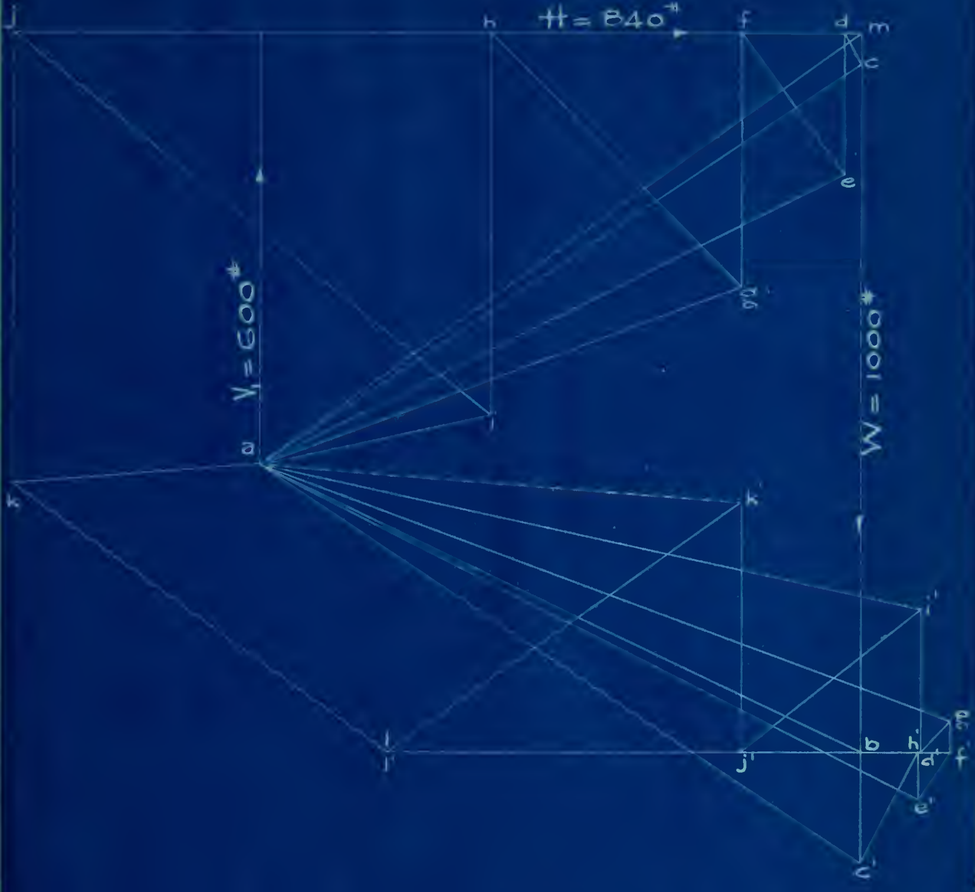


Stress Diagram. Load of 1000# on joint #2.
Scale: 1" = 200#

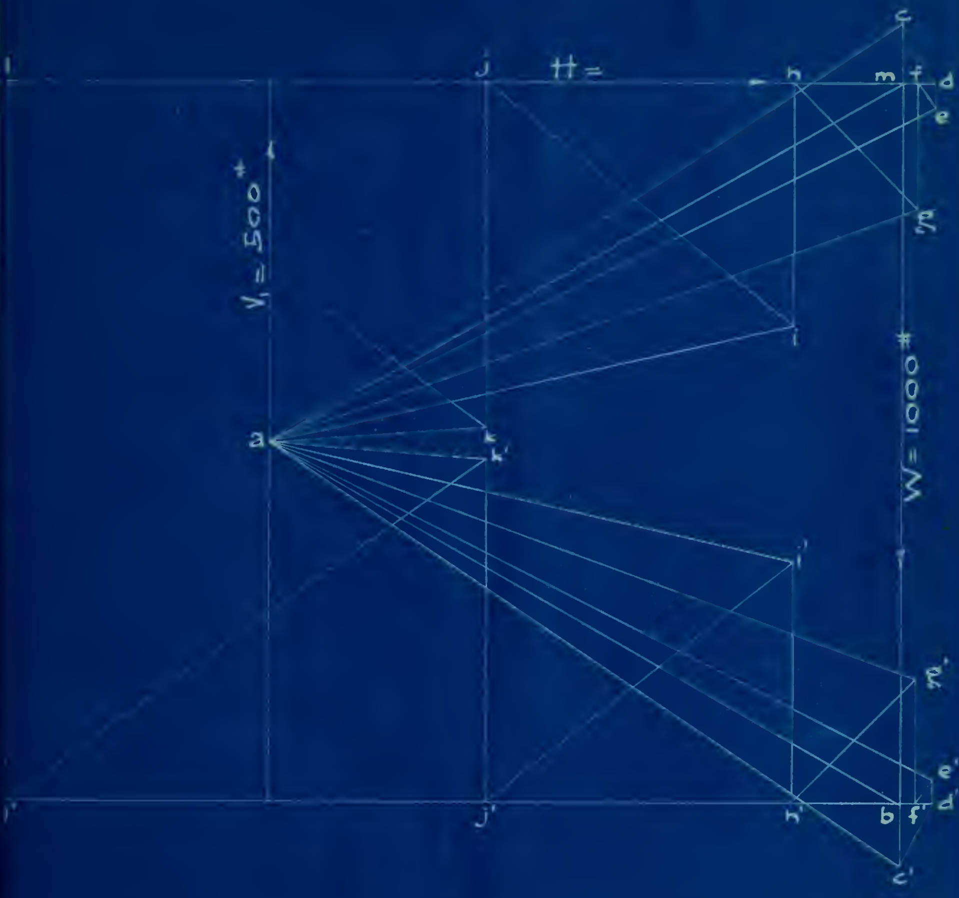


Stress Diagram. Load of 1000* on joint #3.

Scale: 1" = 200*.



Stress Diagram. Load of 1000# on joint #4.
Scale: 1" = 200#



Stress Diagram. Load of 1000# on joint #5.
Scale: 1" = 200#

Final Stresses due to a Single Load of 1000# at each panel point. Areas considered.

Mem.	Joint #1	#2	#3	#4	#5	#6	#7	#8	#9	Σ
BD	- 388	- 250	- 120	- 21	+ 47	+ 80	+ 88	+ 67	+ 40	- 457
BF	- 360	- 737	- 417	- 163	+ 21	+ 127	+ 160	+ 135	+ 80	- 1154
BH	- 297	- 622	- 1011	- 510	- 151	+ 87	+ 175	+ 169	+ 107	- 2035
BJ	- 165	- 406	- 719	- 1176	- 578	- 162	+ 40	+ 102	+ 85	- 2979
BL	- 14	- 115	- 368	- 650	- 1245	- 650	- 368	- 115	- 14	- 3539
AC	- 315	- 602	- 845	- 1008	- 1055	- 1000	- 840	- 600	- 316	- 6581
AE	+ 139	- 280	- 647	- 912	- 1035	- 1040	- 886	- 637	- 340	- 5638
AG	+ 104	+ 251	- 300	- 720	- 958	- 1030	- 918	- 675	- 365	- 4611
AI	+ 36	+ 127	+ 315	- 334	- 748	- 945	- 897	- 683	- 378	- 3507
AK	- 97	- 93	+ 16	+ 340	- 305	- 678	- 745	- 603	- 348	- 2513
BC	- 725	- 466	- 230	- 40	+ 85	+ 150	+ 160	+ 131	+ 74	- 861
DE	- 963	- 675	- 414	- 195	- 35	+ 65	+ 100	+ 91	+ 55	- 1961
FG	+ 64	- 888	- 600	- 350	- 173	- 40	+ 18	+ 30	+ 26	- 1909
HI	+ 93	+ 172	- 770	- 530	- 337	- 195	- 110	- 53	- 18	- 1748
JK	+ 108	+ 207	+ 300	- 625	- 475	- 347	- 243	- 152	- 70	- 1297
CD	+ 823	+ 528	+ 260	+ 48	- 95	- 170	- 180	- 149	- 84	+ 981
EF	- 46	+ 831	+ 510	+ 242	+ 41	- 70	- 120	- 114	- 68	+ 1206
GH	- 89	- 159	+ 845	+ 495	+ 245	+ 55	+ 25	- 48	- 38	+ 1331
IJ	- 161	- 277	- 374	+ 850	+ 545	+ 315	+ 175	+ 85	+ 29	+ 1187
KL	- 185	- 357	- 515	- 645	+ 820	+ 600	+ 418	+ 264	+ 121	+ 521

Final Live Load Stresses
due to a Live Panel-Load of 27000#

Member	Joint #1	#2	#3	#4	#5	#6	#7	#8	#9
BD	-10490	-6750	-3240	-567	+1268	+2160	+2375	+1810	+1080
BF	-9725	-19900	-11250	-4400	+568	+3430	+4320	+3645	+2160
BH	-8025	-16790	-27300	-13770	-4080	+2350	+4725	+4560	+2890
BJ	-4460	-10950	-19410	-31750	-15600	-4375	-1080	+2755	+2295
BL	-378	-3105	-9950	-17550	-33600	-17550	-9950	-3105	-378
AC	-8510	-16250	-22800	-27200	-28550	-27000	-22700	-16200	-8540
AE	+3755	-7560	-17450	-24650	-27950	-28100	-23950	-17200	-9180
AG	-2810	+6775	-8100	-19450	-25900	-27800	-24800	-18270	-9850
AI	-972	+3430	+8500	-9025	-20200	-25500	-24200	-18430	-10210
AK	-2620	-2510	+432	+9190	-8250	-18300	-20100	-16280	-9400
BC	-19590	-12580	-6210	-1080	+2290	+4050	+4325	+3535	+1999
DE	-26000	-18220	-11180	-5270	-946	+1755	+2700	+2455	+1485
FG	+1728	-23950	-16200	-9460	-4675	-1080	+486	+919	+703
HI	+2510	+4640	-20800	-14300	-9110	-5270	-2970	-1430	-486
JK	+2915	+5590	+8100	-16880	-12820	-9375	-6560	-4110	-1888
CD	+22250	+14250	+7030	+1296	-2565	-4590	-4860	-4025	-2270
EF	-1241	+22450	+13770	-6540	+1108	-1890	-3240	-3080	-1837
GH	-2405	-4290	+22800	+13370	+6620	+1485	+675	-1295	-1027
IJ	-4340	-7480	-10100	+22950	+14700	+8510	+4730	+2300	+783
KL	-4990	-9630	-13900	-17410	+22150	+16200	+11300	+7130	+3250

Final Live Load Stresses due to Excess Load of 35000#

Member	Joint #1	#2	#3	#4	#5	#6	#7	#8	#9
BD	-13550	- 8750	-4200	- 725	+ 1640	+ 2800	+ 3080	+ 2350	+ 1400
BF	-12580	-25800	-14600	- 5700	+ 735	+4450	+ 5600	+4730	+ 2800
BH	-10400	-21800	-35400	-17900	- 5300	+ 3450	+ 6130	+ 8920	+3740
BJ	- 5770	-14200	-25100	-41200	-20200	- 5660	+ 1400	+ 3570	+2970
BL	- 490	- 4025	-12900	-22800	-43600	-22800	-12900	-4025	- 490
AC	-11000	-21200	-29600	-35400	-37000	-35000	-29400	-21900	-11070
AE	+4870	- 9800	-27600	-31900	-36300	-36400	-31000	-22200	-11900
AG	+3640	+ 8800	-10500	-25200	-33500	-36100	-32100	-23600	-12750
AI	+1260	+4450	+11000	-11700	-26200	-33100	-31300	-23800	-13200
AK	-3390	-3250	+ 560	+11900	-10660	-23700	-26000	-21100	-12200
BC	-25400	-16250	- 8050	- 1400	+ 2970	+ 5250	+ 5600	+ 4580	+ 2590
DE	-33700	-23600	-14500	- 6830	- 1225	+ 2280	+ 3500	+ 3180	+ 1930
FG	+ 2240	-31000	-21000	-12250	- 6050	- 1400	0	+ 1185	+ 910
HI	+ 3260	+ 6020	-26900	-18500	-11800	- 6830	-3850	- 1850	- 630
JK	-3780	+ 7250	+10500	- 21900	-16600	-12150	- 8500	-5330	-2450
CD	+28800	+18500	+ 9100	+ 1680	- 3325	- 5950	- 6300	- 5220	- 2940
EF	- 1610	+29100	+17800	+ 8470	+ 1435	-2450	- 4200	- 3990	- 2380
GH	-3110	- 5570	+29600	+17320	+ 8575	+1925	+ 875	- 1680	- 1330
IJ	- 5640	- 9700	-13100	+29750	+19100	+11030	+ 6120	+ 2980	+ 1020
KL	-6470	-12500	-18000	-22600	+28700	+21000	+14650	+ 9240	+ 4240

Final Stresses (Max)

Member	Dead	+ Live	- Live	Impact	Temp.	Wind	Totals.	
							With wind	Without wind.
BD	- 9140	9065	24107	-18300	± 6400	-16085	-148532	-57947
BF	-23080	14763	51175	-38900	15200	-27913	-288268	-128355
BH	-40700	15625	78065	-56100	27150	-32874	-408829	-207015
BJ	-59580	5050	97075	-63200	39400	-33059	-490714	-259255
BL	-70780	0	105566	-62100	45500	-20480	-511426	-283946
AC	-131620	0	186200	-110500	21400	-59711	-533231	-449720
AE	-112780	3755	164340	-102400	27150	-45013	-497683	-406670
AG	- 92220	9555	142420	-93000	35200	-37900	-460040	-362840
AI	- 20140	15402	115165	-77900	46000	-33035	-419740	-309205
AK	-50260	12332	84130	-54700	57600	-28000	-356690	-246690
BC	-17220	17465	45270	-34400	11900	-21022	-129812	-108790
DE	-39220	7195	69316	-49800	12150	-42625	-213111	-170486
FG	-38180	4348	62415	-44900	12030	-22920	-180445	-157525
HI	-34960	8530	60466	-39400	9820	-20375	-165021	-144646
JK	-25940	19005	56653	-38800	4110	-11900	-136900	-125000
CD	+19620	51376	19750	+39000	13480	+34150	+157626	-123476
EF	+24120	50518	12248	+38300	15000	+21863	+149800	-127938
GH	+26620	51750	10297	+37200	17000	+11650	+144220	-132570
IJ	+23240	60773	24920	+41700	15700	+10368	+152281	-141913
KL	+10420	66580	51120	+50600	7320	+15822	+150742	-134920

Summary of Stresses, Sections, & Weights of Members.

Member	DEAD Prel. & Final	LIVE Prel.	LIVE Final	TEMP. Prel.	TEMP. Final	MAX.* Stress	Lengths of Members	SECTIONS	Areas.	WEIGHTS
BD	-9140	+7828 -25115	+9065 -24107	±9780	±6400	-153100 w	21'-0"	1-18"×3/8" Cov. Pl. 2-12"-20.5# [s	8.87	
BF	-23080	+11368 -53540	+14763 -51175	24600	15200	-295650 w	21'-0"	1-18×3/8" Cov. Pl. 2-12"-25# [s	18.9	
BH	-40700	+10370 -83925	+15625 -78065	41500	27150	-416640 w	21'-0"	1-18"×3/8" Cov. Pl. 2-12"-35# [s	27.6	
BJ	-59580	+3270 -108425	+5050 -97075	60500	39400	-493250 w	21'-0"	1-18×3/8" Cov. Pl. 2-12×3/8" Side Pl.	31.0	
BL	-70780	0 -119732	0 -105566	69700	45500	-511426 w	21'-0"	2-12"-30# [s	32.2	38925*
AC	-131620	0 -177450	0 -186200	33800	21400	-449720	25'-2 21/32"	1-18×3/8" Cov. Pl. 2-15"×50# [s	36.6	
AE	-112780	+4220 -153735	+3755 -164340	41500	27150	-501560 w	23'-7 23/32"	2-15"-55# [s	32.7	
AG	-92220	+11280 -130180	+9550 -142420	53800	35200	-464825 w	22'-4 21/32"	2-15"-50# [s	29.9	
AI	-70140	+17780 -102120	+15420 -115165	70500	46000	-427450 w	21'-6 1/8"	2-15"-45# [s	27.1	
AK	-50260	+16650 -67355	+12332 -84130	88000	57600	-362850 w	21'-0 11/16"	2-15"-40# [s	22.7	45800*
BC	-17220	+14018 -47130	+17465 -45270	18150	11900	-117520	53'-11"	2-15"-35# [s	18.5	
DE	-39220	+6240 -70470	+9195 -69316	18550	12150	-174083	39'-11 3/8"	4 Ls 6"×4"×1/2"	17.2	
FG	-38180	+2541 -65700	+4348 -62415	18400	12030	-159700	29'-1"	4 Ls 6"×4"×7/16"	15.7	
HI	-34960	+6720 -64875	+8530 -60466	15000	9820	-148900	21'-3 7/8"	4 Ls 6"×4"×7/16"	14.6	
JK	-25940	+15555 -58060	+19005 -56653	6420	4110	-134500	16'-8"	4 Ls 6"×4"×3/8"	13.1	34000*
CD	+19620	+53212 -21700	+51376 -19750	20600	13480	+133350	45'-1 9/16"	4 Ls 5"×3 1/2"×3/8"	8.3	
EF	+24120	+53585 -9490	+50518 -12248	22900	15000	+134060	35'-10 15/32"	4 Ls 5"×3 1/2"×3/8"	8.4	
GH	+26620	+55262 -8491	+51750 -10297	25900	17000	+137725	29'-11 5/32"	4 Ls 5"×3 1/2"×3/8"	8.6	
IJ	+23740	+65634 -22970	+60773 -24920	24250	15700	+154375	26'-9 3/4"	4 Ls 5"×3 1/2"×7/16"	9.6	
KL	+10420	+68852 -49675	+66580 -51120	11150	7320	+160500	25'-10 1/2"	4 Ls 5"×3 1/2"×7/16"	10.0	-
LL'	-20000	-64800	-64800	0	0	-84800	15'-1 13/32"	4 Ls 5"×3 1/2"×3/8"	7.6	30000*

Stringers, Floorbeams, Ties, Rails, Fastenings, etc. = 136400#

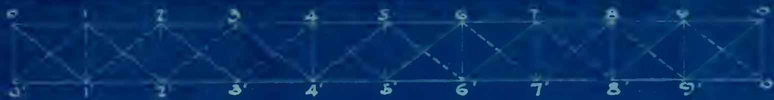
Portal, Lateral Systems, & Sway Bracing = 53500#

Rivets, Connection Plates & Lace Bars = +15% = 50800#

Total Weight of Arch = 389425#

*Max stress + Impact + Reversal + W. Wind.

Design of Upper Lateral System.



Wind + lateral force = (200# + 10% 2500#) 21 = 9450# = panel load

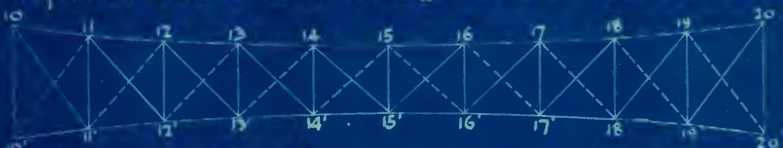
Stresses. (Live Load)

0-1 = BD = -74500#	0-1' = +85,600#	Use 2-6"×3½"×⅜" Ls.
1-2 = BF = -132,000#	1-2' = +68400#	" 2-4½×3×⅜" Ls.
2-3 = BH = -174,000#	2-3' = +53,400#	" 2-3½"×3"×⅜" Ls.
3-4 = BJ = -198,000#	3-4' = +40,000#	" 2-3½×3"×⅜" Ls.
4-5 = BL = -207,000#	4-5' = +28,600#	" 2-3½"×3"×⅜" Ls.

Max. stress of -20250# in struts taken care of in design of Floor beams.

Design of Lower Lateral System.

Wind panel load = 200# × 22½' = 4500# (horizontal)



Stresses. (Live load)

10-11 = AC = -23,800#	10-11' = +29,900#	10-10' = -20,250#
11-12 = AE = -46000	11-12' = +23,900	11-11' = -18,000#
12-13 = AG = -65300	12-13' = +17,800	12-12' = -13,500
13-14 = AI = -77500	13-14' = +11,000	13-13' = -9,000
14-15 = AK = -82,000	14-15' = +3,735	14-14' / 15-15' = -4,500

For all diagonals and struts (except 10-10') use 4-3½"×3"×⅜" (laced).

Total stress in 10-10' = 20250# + 60750# (from portal) = -81,000#

Use 4-5×3½"×⅜" Ls (Long legs outstanding).

DESIGN OF STRINGER

Span = 21'-0". Stringers 6'-0" center to center.

Dead Load due to 6"×10"×10'-0" Ties = 2360#
" " " " rails, fastenings, etc. = 1575
" " " " stringer = 2100
Total = 6035#

Max. End Shear D.L. = 3000
" " " L.L. = 27850
Impact = 26050 Total shear = 56900#
" D.L. Bending M. = 191000 in. lbs.
" L.L. " " = 1521000 " "
Impact = 1422000 " "

Total Bending Moment = 3134000 in. lbs.

Use a 24"×3/8" Web & Flanges of 2-6"×6"×1/2" Ls.
Rivet Spacing over 20" panels beginning from end is $2\frac{3}{4}$", 3", $3\frac{1}{2}$", $4\frac{1}{8}$" & 6".

DESIGN OF FLOOR-BEAM.

Span = 12'-0"

Dead Load Concentration = 6750#
Live " " = 34600
Impact = 30200
Max. End Shear = 71550

Max. Bending Moment = 2,580,000 in. lbs.
Use web = 36"×3/8". Flanges of 2-6"×4"×7/16" Ls.
Space rivets $3\frac{1}{2}$" between stringer & chord.
" " 6" " stringers.

SWAY BRACING.

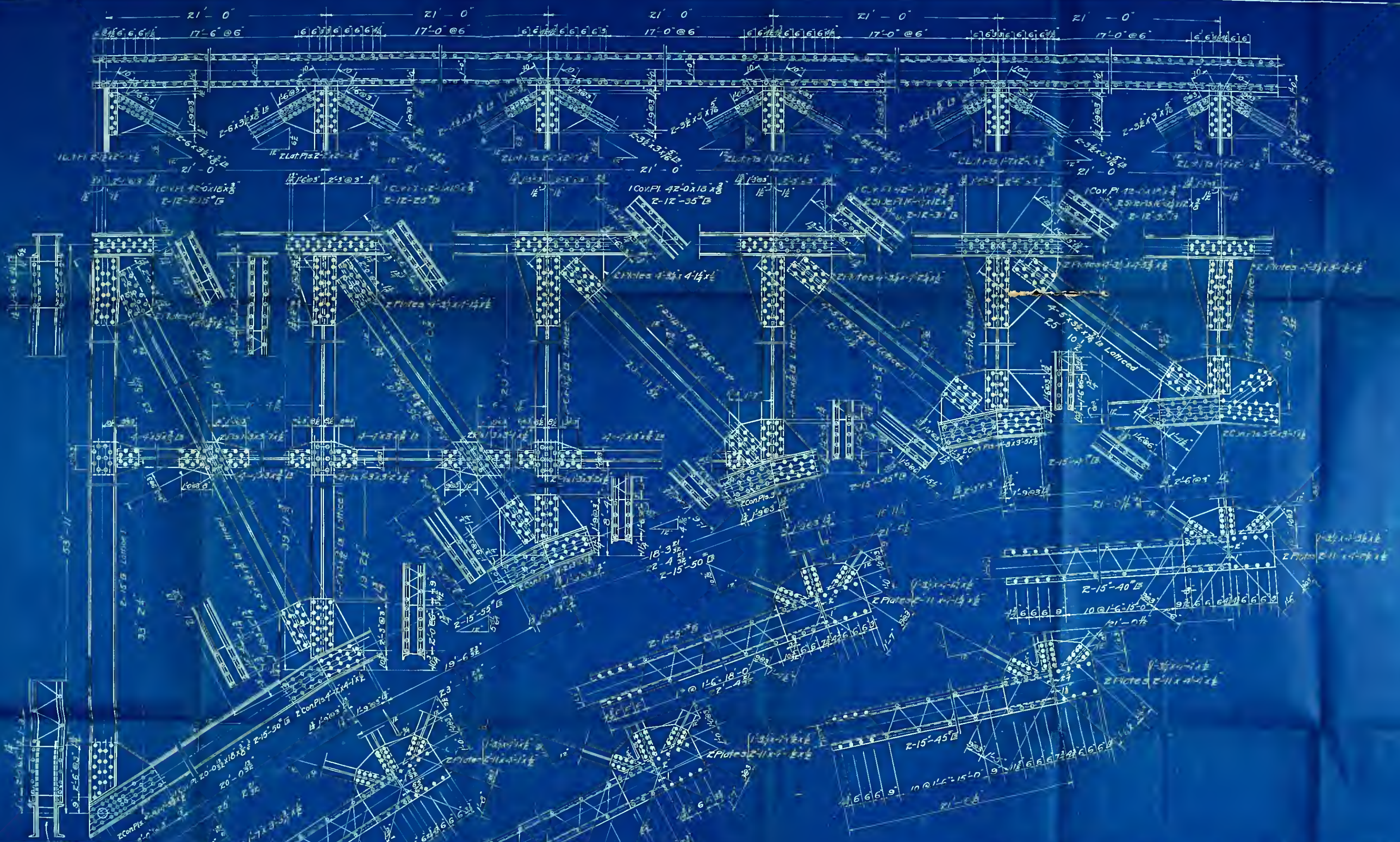
Max. stress in diagonals = +23800#
" " " struts = −13950#
For diagonals use $3\frac{1}{2}$"×3"×3/8" Ls (in pairs)
" struts use 4-$3\frac{1}{2}$"×3"×3/8" Ls.

PORTAL BRACING

Max. stress in diagonals = +92000#
" " " struts = −47250 + −13500# (from girder) = −60,750#
For diagonals use 4-$3\frac{1}{2}$×3×3/8 Ls
" struts use 4-4×3×3/8 Ls. $\frac{l}{r}$ = 120.

PIN AT ABUTMENT HINGE.

Max. M = 859,000 inch lbs. Use a $7\frac{1}{4}$" dia. pin.



ARMOUR INSTITUTE OF TECHNOLOGY

CIVIL ENGINEERING DEPARTMENT

THESIS

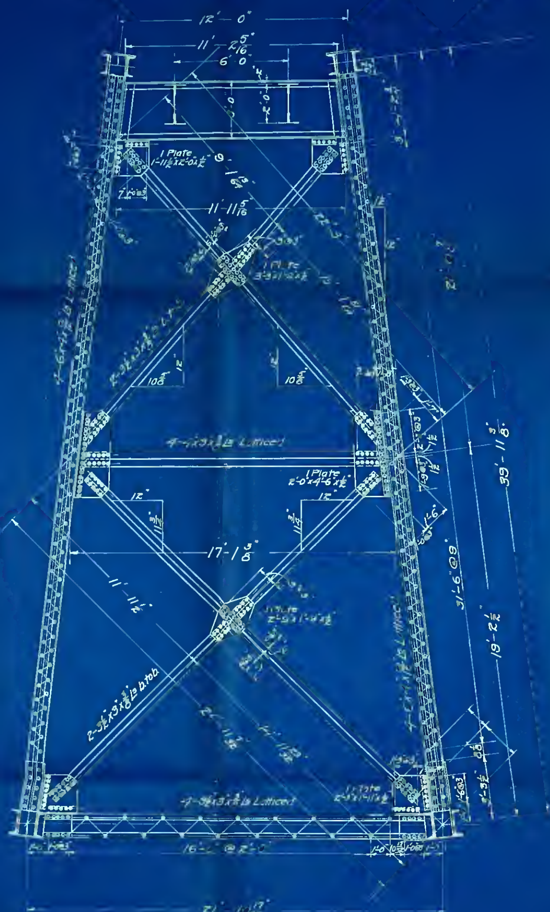
SPANDREL-BRACED TWO-HINGED ARCH

SPAN = 210'-0"

SCALE

SUBMITTED BY

MAY 1911

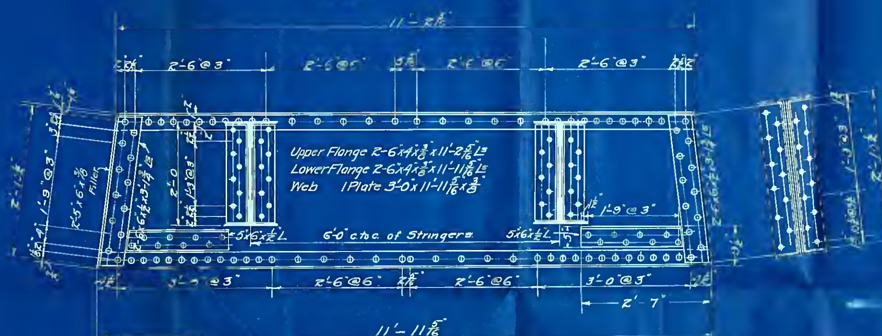


Cross Section at First Panel Point

Scale 3/8"=1'

Scale 1"=1'

Stringer



Scale 1"=1'

Floor Beam

SHEET NO. 2

FLOOR SYSTEM & CROSS BRACING